Bolted beam-column moment connections between cold-formed steel members

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ABSTRACT Twelve beam-to-column connections between cold-formed steel sections consisting of three beam depths and four connection types were tested in isolation to investigate their behavior based on strength, stiffness and ductility. Resulting moment-rotation curves indicate that the tested connections are efficient moment connections where moment capacities ranged from about 65% to 100% of the connected beam capacity. With a moment capacity of greater than 80% of connected beam member capacity, some of the connections can be regarded as full strength connections. Connections also possessed sufficient ductility with rotations of 20 mRad at failure although some connections were too ductile with rotations in excess of 30 mRad. Generally, most of the connections possess the strength and ductility to be considered as partial strength connections. The ultimate failures of almost all of the connections were due to local buckling of the compression web and flange elements of the beam closest to the connection.

1 INTRODUCTION

1.1 *Cold-Formed Steel Sections*

Due to their high structural performance and durability, cold-formed steel sections are well suited for building construction, and their light weight is a definite advantage. Of late, their usages as primary structural members have become more widespread. They are increasingly being used as beams and columns in low to medium rise building construction and medium span portal frames. Engineers designing cold-formed steel structures can find guidance in several codes of practice (AISI 1996, AS/NZ 4600 2005, Eurocode 3 1996, BS 5950 1998). Design recommendations on connections among cold-formed steel sections are mostly related to the load carrying capacities of individual fasteners such as bolts, screws, rivets and spot welds. However, guidance on the structural performance of the bolted moment connections for cold-formed steel sections is lacking.

1.2 *Connections between Cold-formed Steel Sections*

Practically, joints are designed as either pinned or rigid. Pinned connections are connections where only the end shears of the connected beams are transferred from beams to columns. They possess insufficiently low stiffness and thus are incapable of transmitting moments at the ultimate limit state. Beams designed assuming pin-ended connections are conservatively sized, and columns are designed only for axial load and 'eccentric' moments, thus resulting in lighter column sections. Full strength or rigid connections are assumed to be able to fully transfer end moments from beam to column. The rotational stiffness of the connection is high and mid-span design moments of beams are significantly reduced, resulting in lighter beam sections. However, the resulting columns, designed for both the axial load and the end moments, are heavier sections. This does not represent the true behavior of joints where true joint behavior lies somewhere between these two extremes. True joint behavior can be observed through moment-rotation curves of connections and most would show that their behaviours are semi-rigid. Connections with semi-rigid behavior are termed as semi-rigid or partial strength connections.

Dubina & Zaharia (1997), Dubina (2008), Lim & Nethercot (2002, 2003 &2004) Chung & Lau (1999), Wong & Chung (2002) and Yu & Chung (2005) reported experimental and analytical testing on bolted moment connections for cold-formed double channel sections. Dundu & Kemp (2006) tested back to back eaves bolted connection between cold-formed sections while Mills & LaBoube (2002) tested similar back to back connections for portal frames using self-drilling screws. Most the researches aforementioned concentrated on the moment carrying capacity of the connections with the aim of achieving the most efficient connections when compared to the capacity of connected beam members. Most connections except those by Dundu & Kemp (2006) and Mills & LaBoube (2002) were achieved using interconnectors such as hot-rolled angles and gusset plates.

1.3 *Partial Strength Moment Connections*

A partial strength moment connection possesses some degree of rotational stiffness and moment capacity, but is insufficiently stiff to develop full continuity and unable to achieve full moment capacity of the members at a joint, as shown in Figure 1. EC3 Part 1-8 (2002) recognizes the use of partial strength connection and includes specific rules to quantify different types of connection based on their strength (moment resistance), rigidity (rotational stiffness) and ductility (rotational capacity). EC3 Part 1-1 (2005) allows frames to be designed as semi-rigid using partial strength connections, provided that the moment resistances of connections are quantifiable and the ductility of the connections ensured.

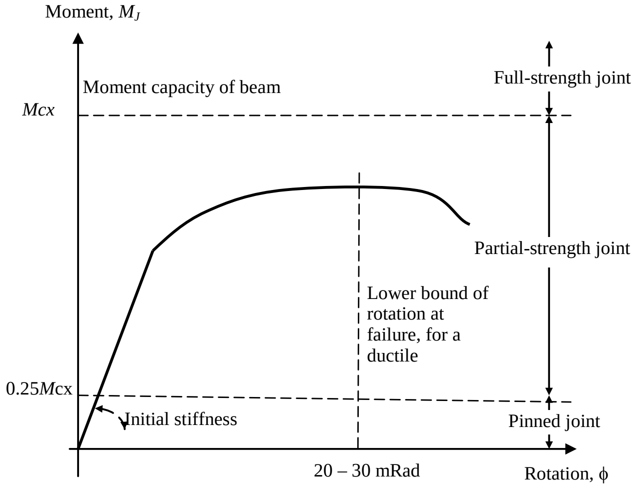


Figure 1. Moment-rotation relationship of a partial strength (semi-rigid) connection

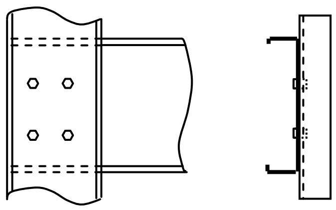
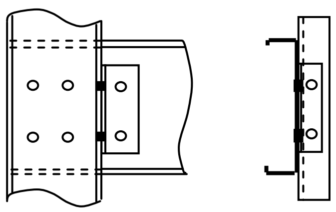
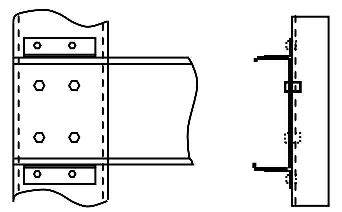
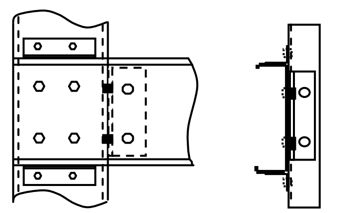
2 EXPERIMENTAL INVESTIGATION

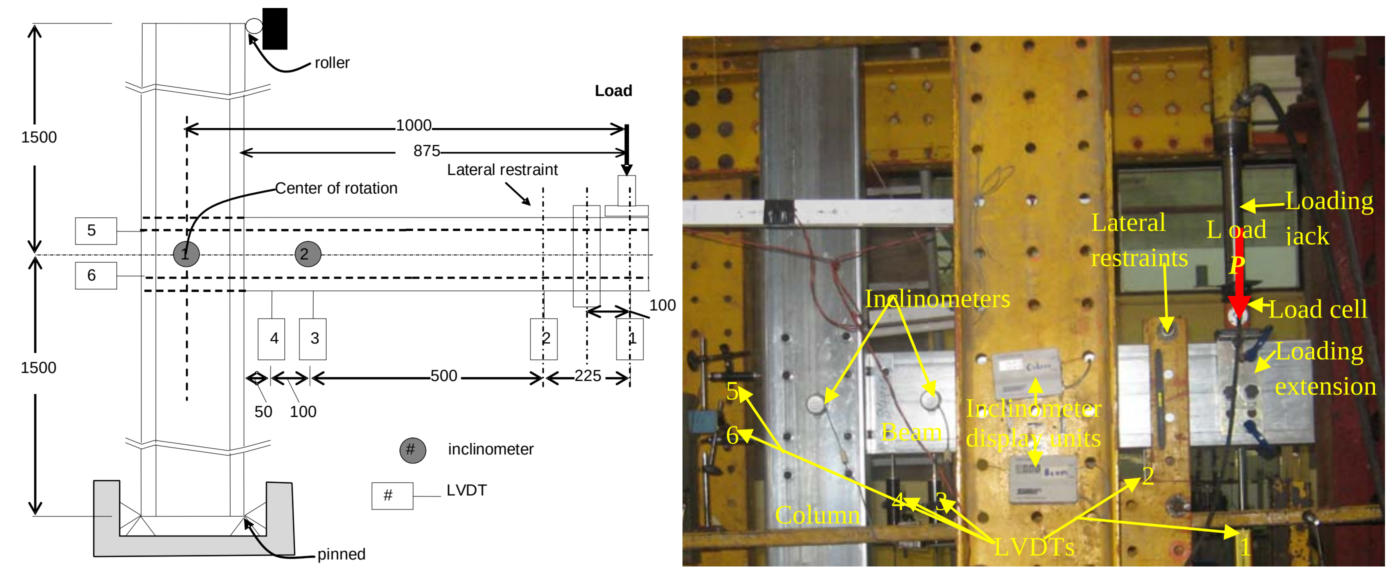
This experimental investigation is carried out in order to assess the structural performance and suitability of proposed bolted moment connections between cold-formed section beams and columns as partial strength connections with a view for practical application.

2.1 *Proposed Connections*

Proposed connection configurations are as shown in Table 1. Beam and column used were lipped C-sections of grade G350; connected back-to-back (web-to-web) without any interconnector between the webs of beams and columns. Four connection

Table 1 Configuration of proposed connections

Connection Type	Side and front profile	Front and back view of assembled connections	
(a) Bolt only connection(T1) 4 bolts used			
(b) Angle web cleat (T2) 8 bolts used			
(c) Header & seater connection (T3) 12 bolts used			
(d) Angle web cleat plus header & seater connection (T4) 16 bolts used			



(a) Schematic Diagram

(b) Actual Testing in progress

Figure 2. Test Configuration and Instrumentation

configurations were tested with cold-formed steel angles of similar grade and thickness used as stiffeners and arranged in various configurations. Beam depths of 150, 200 and 250 mm were used while the columns used were kept the same at 250 mm deep sections. M12 8.8 bolts were used to connect the beams to columns while the bolt hole clearance was 0.5 mm. Four types of connections were tested for each beam depth. With three beam depths and four types of connection for each beam depth, a total of twelve connections were tested in this investigation. With the increase in the number of bolts making up the connections, fabrication was more difficult and the cost of the connections increased.

2.2 *Test Configuration and Instrumentation*

Figures 2(a) and (b) show the overall test configuration used in the test showing beam, column and loading position, and also positions of all LVDTs and inclinometers. Six LVDTs were used in all the tests. LVDTs 1-4 were located along the bottom flange of the beam to measure the vertical displacements; and LVDTs 5-6 on the side of the column to detect any horizontal displacement of the column. The inclinometer on the beam measured the rotation, ϕ_B, against the applied loading. The other inclinometer was placed at the centre of the column shear panel, which was also the centre of rotation, thus providing the rotation of the column, ϕ_C. The true rotation of the joint ϕ is the difference between ϕ_B and ϕ_C. The load was applied to the beam by using a single hydraulic jack and was measured by a 300 kN load cell. All LVDTs, and load cell were connected directly to a computer controlled data logger. All the readings monitored were transferred from the data logger and recorded and stored in the hard disk of the computer. Readings from the inclinometers were recorded manually via the digital display unit of the Lucas rotational inclinometers.

3 RESULTS AND DISCUSSION

From the moment and rotation data obtained from the tests, moment versus rotation curves were plotted. Moment-rotation curves reveal the behavioral characteristics of a connection. The moment resistance (strength), stiffness (rotational rigidity) and the rotational capacity (ductility) can be determined from the moment-rotation curves. The moment resistance of a connection, M_{ult}, is the ultimate moment of a connection while the rotational capacity is the relative rotation of the connection at failure. The stiffness of a connection is the slope of the moment-rotation curve in the linear part of the curve. A ratio of the ultimate moment over the moment capacity of the connected beam is then calculated to classify the connection in accordance to their strength classification.

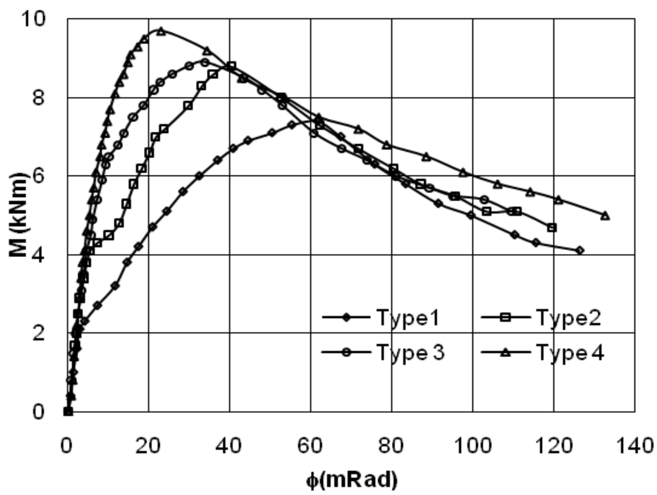
Figures 3(a-c) show the plotted moment-rotation curves for all connection types grouped according to beam depth. Table 2 summarizes the values deduced from M-ϕ curves. It can be seen that as the beam depth increases, M_{ult}, also increases. This is expected since deeper beams have higher moment capacities. Almost all connection failures (except for T1 of 250 mm deep beam which failed due to the tilting of bolts) were due to the buckling of the compression web and associated sympathetic compression flange buckling. M_{ult} also varies according to the connection type. T1 has the least ultimate moment while T4 has the highest ultimate moment. M_{ult}, of T2 is higher than T3 although T3 uses more bolts and this can be attributed to the difference in the placement of angle stiffeners.

The initial stiffness of the connections is almost equal for all the connection types. The difference is in the instance at which the initial stiffness starts to change and the gradient of the M-ϕ starts to decrease indicating loss of stiffness. The base T1 connection with the least number of bolts starts to lose the stiffness the earliest followed by T2, T3 and T4 connections. At the instance when the gradient changes is when the applied moment (load) exceeds the resistance provided by the frictional force between the tightened bolts and washers with the surface of the connected member. With more bolts used in connections from T1 to T4, the greater is the resistance. Prior to the change in gradient, the stiffness is provided by the beam cross-section itself while after the change the stiffness is provided by bearing between bolt shanks and connected member material. Within similar connection types, initial stiffness is significantly increased by increasing beam depths.

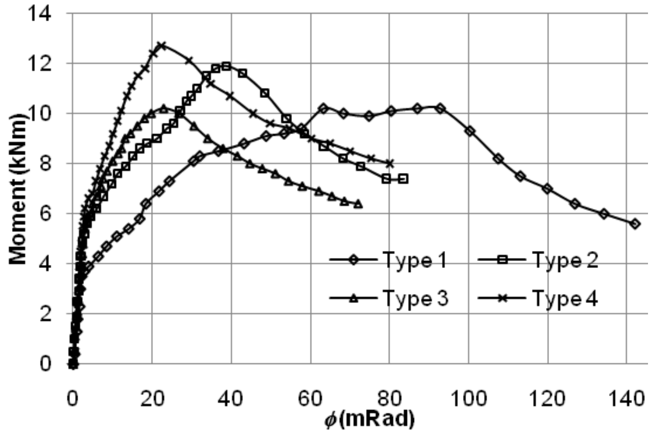
Beam depth does not affect the ductility significantly. Similar connection types across all three beam depths exhibit almost equivalent rotational capacity at failure. Ductility of the connections is more affected by the connection type with T1 connection being the most ductile and T4 connection the least. Placement of the angle stiffener between the beam web and column flange for T2 connection not only result in a higher M_{ult} compared to T3 connection but it also has a larger rotational capacity (more ductile) than T3 connection.

The ratio of ultimate moment to the reduced moment capacity of connected beam M_{ult}/M_{cxr}, ($M_{cxr}=P_yZ_{xr}$; where Z_{xr} is the reduced elastic modulus to cater for the effects of local buckling), shows that all connections possess ample moment capacity and are efficient as moment connections. Only the T1 250 mm deep beam connection failed due to connection failure; i.e. tilting of bolts (Figure 4(f)). The lowest is more than $0.71M_{cxr}$ and the highest is about $1.0M_{cxr}$. The 150 mm deep beam is the most efficient as its shallow web will give a smaller depth over thickness ratio thus making it less prone to web local buckling.

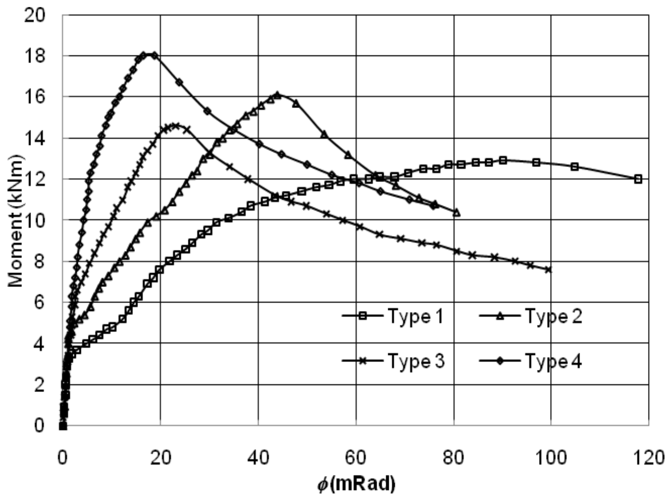
Some of the connections can be regarded as partial strength connections based on guidelines given in Figure 1. In terms of M_{ult}/M_{cxr} ratio all connections exceed the $0.25M_{cxr}$ ratio. All T4 and T2 connections, and T3 connections of 150 mm beam exceed $0.8M_{cxr}$, and these connections can be considered to be almost rigid. But when considering



(a)150 mm deep beam



(b) 200 mm deep beam



(c) 200 mm deep beam

Figure 3. Moment-rotation curves of tested connections grouped according to beam depth.

Table 2 Summary of values deduced from plotted moment-rotation curves

Connection Type	Beam depth (mm)	Initial stiffness (kNm/mRad)	Ultimate moment M_{ult} (kNm)	Rotational capacity (mRad)	Beam reduced moment capacity M_{cxr} (kNm)	M_{ult}/M_{cxr}
	150	0.76	7.4	61.46	9.72	0.76
T1	200	1.63	10.2	63.20	14.36	0.71
	250	3.08	12.9	90.09	18.88	0.68
	150	0.98	8.8	40.33	9.72	0.91
T2	200	2.22	11.9	38.76	14.36	0.83
	250	3.45	16.1	43.83	18.88	0.85
	150	0.94	8.9	33.87	9.72	0.92
T3	200	1.99	10.2	22.87	14.36	0.71
	250	2.99	14.6	23.05	18.88	0.77
	150	0.98	9.7	22.87	9.72	1.00
T4	200	2.26	12.7	22.34	14.36	0.88
	250	3.48	18.0	18.68	18.88	0.95

ductility most of the connections are too ductile. All T1 and T2 connections had a rotational capacity of over 30 mRad at failure load and are too ductile to be considered as partial strength connections. Only T3 and T4 connections can be considered to possess the necessary ductility to be considered partial strength connections.

All the connections tested failed in a similar manner except for the T1 250 mm deep beam that failed due bolt tilt as seen in Figure 4 (f). The progression of failure was initiated by the formation of a dent at a location nearest to the bottom inner bolt as the applied load approached failure. Failure occurred when a crease formed, which initiated from the bottom web-to flange bend and progressed upwards as failure progressed. This was accompanied by the almost instantaneous sympathetic buckling of the bottom compression flange. The connections at failure are as shown in Figures 4(a)-(d). Figure 4(e) shows the buckling of the bottom compression flange at the failure of the connection. This progression towards failure is reflected in the moment-rotation curves by the reduction in the slope of the curves as the failure load was approached.

4 CONCLUSIONS AND RECOMMENDATION

Twelve bolted moment connections were tested in isolation in order to observe their behavior in terms of moment capacity, stiffness and ductility. Almost all are efficient in terms of their moment carrying capacity where their moment capacity range was between 70 to 100 percent of the connected member capacity. The failures modes were ductile where there were no sudden or catastrophic failures.

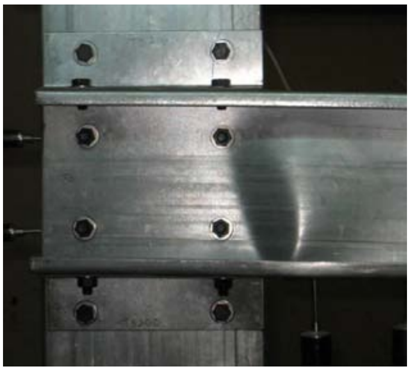
The range of rotational capacity was from 63 mRad for T1 200 mm deep connection to a minimum of about 19 mRad for T4 250 deep beam connection. T1 250 mm deep connection was discounted due to the failure of the connection itself which is undesirable. Other connections failed by local buckling of the compression web closest to the connection itself.

All T3 and T4 connections can be considered as partial strength connections. Their moment capacities are less than but are in excess of 25 percent of the moment capacity of the connected member. T1 and T2 connections are overly ductile and would produce excessive rotation outside of the recommended rotational capacity of between 20 to 30 mRad. The limits though, were established considering hot-rolled sections. Excessive rotation (deformation) is a common problem when considering connection between cold-formed sections. Chung (1999) faced similar excessive rotations and limited rotation to 50 mRad and moment capacity of connection was considered at that particular rotation. With that same view and considering partial strength connections, rotational capacity should be limited at 30 mRad. With the moment capacity ratio of the connections being far in excess of the 25 percent lower limit, this rotational capacity limit is practical.

For connections that have excessively high moment capacity ratio, smaller sized bolts should be considered which will reduce the moment capacity of the connections. This will result in lower moment capacity ratio and result in connections that are more fitting to the partial strength analogy.

(a) T1 200 mm deep beam connection

(b) T2 200 mm deep beam connection



(c) T3 200 mm deep beam connection

(d) T4 200 mm deep beam connection

(e) Buckling of bottom flange

(f) Bolt tilt failure of T1 250 mm deep beam

Figure 4. Failure modes of connections

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